

SHIP IMPACT PROTECTION DESIGN FOR THE INCHEON BRIDGE

Lars Hauge, International Bridge Director, COWI

Henrik Gluwer, Risk Analysis Specialist, formerly COWI - now Det Norske Veritas

Oliver Kübler, Risk Analysis Specialist, COWI

Jørgen S. Steenfelt, Geotechnical Specialist, COWI

Jens-Erik Jepsen, Civil Engineer, COWI

Erik Yding Andersen, Civil Engineer, COWI

Abstract : *This paper discusses and presents the application of a risk based design approach in the detailed design of the ship impact protection for the Incheon Bridge. An important part of this approach was to apply a credible modelling of the structural behaviour of the Ship Impact Structures / Dolphins during an impact. Hence detailed assessments of the geotechnical profiles were performed, accurate FE-models were established and tested in ABAQUS against the results of physical model tests from a large scale centrifuge model of an idealized prototype model of the dolphin.*

INTRODUCTION

The Incheon Second Bridge in South Korea will be crossing the navigational channel used by the ship traffic calling Incheon Harbour. The bridge features an 800m main span providing a horizontal navigational clearance of 625m to ensure safe passage for the ship traffic.

But the possibility of accidental ship impact will always exist for a bridge spanning navigated waters and to safeguard towards this hazard, the design includes ship impact protection of the four piers on either side of the navigation channel - see Figure 1.



Figure 1: Illustration of the tender design of the Second Incheon Bridge.

To ensure an appropriate and adequate final design of the ship impact protection, specific design requirements are included in the Project Performance Requirements (PPR). In short the requirements states that:

- The eight piers E1-E4 and W1-W4 shall be protected towards an accidental impact from a 100,000 DWT ship
- The protection and/or impact capacity of the approach span piers (E10-E5 and W5-W10) shall be determined by risk analysis in accordance with the method given in the AASHTO LRFD Bridge Design Specifications
- In the risk analysis the Incheon Bridge shall be considered a critical bridge implying that the frequency of accidental collapse due to ship impact must not exceed 10^{-4} per year

RISK BASED DESIGN

Risk analysis is used in the design process when the design - in this case a bridge structure - can be exposed to rare hazards with large consequences. Ship impact to a bridge from larger vessels is such a rare hazard which may have catastrophic consequences to the bridge. A risk analysis is used to identify the hazards, calculate the frequency with which the hazards are expected to occur and determine the consequences to the bridge structure if they do occur.

Quantification of the risk in terms of frequency and consequences provides the basis for making informed decisions about the appropriate level of protection towards the hazard.

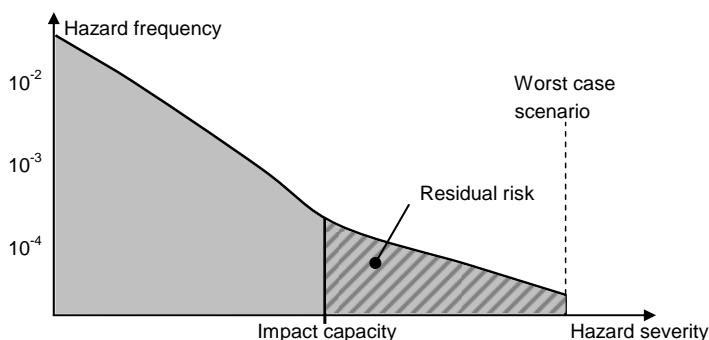


Figure 2: Relation hazard frequency - severity.

When an impact capacity has been chosen (e.g. a specific impact capacity of pier and dolphin) the residual risk or just "the risk" is the frequency of hazard scenarios for which the capacity is known to be insufficient (e.g. the impact capacity is smaller than load effects from the ship impact). So a higher capacity will result in a smaller residual risk.

When a worst case scenario can be defined the use of a capacity that can resist this scenario will result in zero residual risk.

The general hazard of ship impact to the Incheon Bridge involves a multitude of hazard scenarios that have to be considered to ensure an acceptable ship impact risk for the bridge as a whole - see Figure 3.

Each scenario contributes to the frequency of bridge collapse and the idea in a risk based design against ship collision is to identify the potential risk contribution from each of the many scenarios and to iteratively determine appropriate design requirements (i.e. impact capacities and protection configuration) to the impacted piers and girders that will ensure that the total risk - i.e. the sum of all individual contributions - does not exceed a predefined accepted level. This offers an opportunity for optimization of the design requirements such that the risk is reduced for elements and scenarios where the risk reduction is practicable and most cost-efficient.

The optimization will naturally focus on the most exposed elements, and for the Incheon Bridge this implies the piers closest to the main span. Initially the iterative optimization will normally aim at ensuring an even distribution of the residual risk since this will ensure a protection in immediate balance with the exposure. Subsequent considerations on cost, practical conditions, construction methods and construction efficiency may suggest modification in the design that will lead to a more uneven distribution of the residual risk. But the risk based design method can easily accommodate such considerations and restrictions as long as the total risk remains below the accepted level.

AASHTO LRDF Bridge Design Specification Method

The AASHTO risk calculation method only represents pier impacts in a direction parallel with the navigational channel and does thus not provide the full detail of the scenario breakdown shown above. The scenarios explicitly considered in the AASHTO are highlighted in the Figure 3.

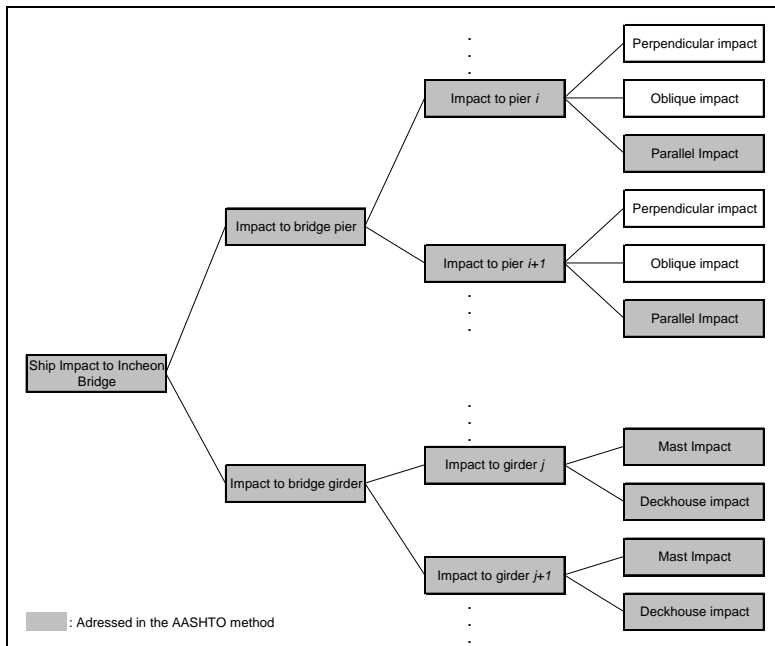


Figure 3: Hazard scenarios for the Incheon Bridge, hazard scenarios considered by AASHTO.

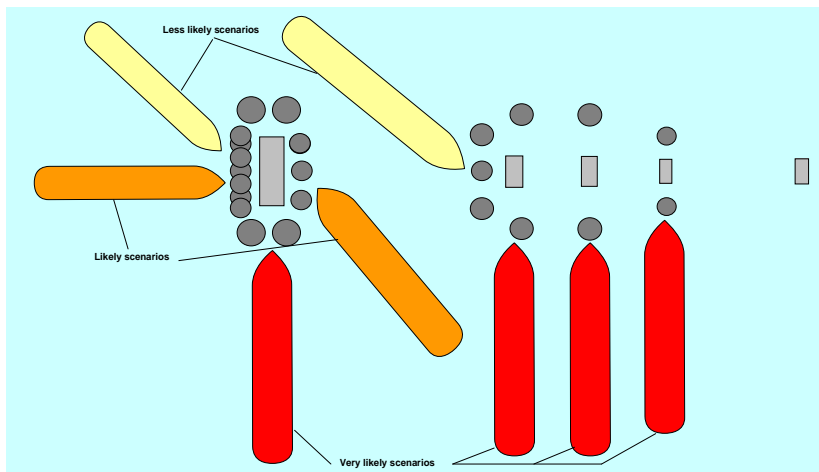


Figure 4: Impacts considered by AASHTO - red
Less likely impacts - yellow & orange.

Impacts parallel to the bridge are implicitly addressed in AASHTO by specifying a 50% impact load in this direction. But the associated risk is not calculated and separate optimization on this specific scenario cannot be made.

Most often this limited detail of the method is not a concern. But when external protection is used the risk analysis will not capture and evaluate the variation of the protection efficiency at different impact angles. Optimization of the protection design for less likely impacts (see Figure 4) will therefore require a more detailed representation than used in the AASHTO method.

With its simplified representation of impact scenarios the AASHTO method will be applicable to determine the pier impact capacities of unprotected piers and to determine the initial requirements to the protection for the most likely perpendicular impact direction towards the protected piers.

APPLICATION OF THE AASHTO METHOD

The utilization of the AASHTO method as required in the PPR is presented and discussed in the following.

METHOD DISCUSSION

Geometric extent of risk analysis

The risk calculation in the AASHTO Method II only includes the parts of the bridge that are within a distance of $3 \times L_{OA}$ from the transit vessel path (=centre of the ship navigation lanes). For the Incheon Bridge this implies that only the 8 central piers will be included. According to the AASHTO method the resistance of the remaining piers and girders (approach bridges) shall be based on the defined minimum impact requirements.

Representation of external protection

The AASHTO method does not explicitly model the effect of external protection but assumes that the capacity of the external protection can be applied as if it represented the pier impact capacity. To be able to utilize the actual capacity of the protected pier in parallel with the protection capacity, it is necessary to represent the external protection as an additional element in the AASHTO calculation method.

This is done by representing the protection capacity in terms of the collision energy it is able to dissipate in an impact as shown in Figure 5 :

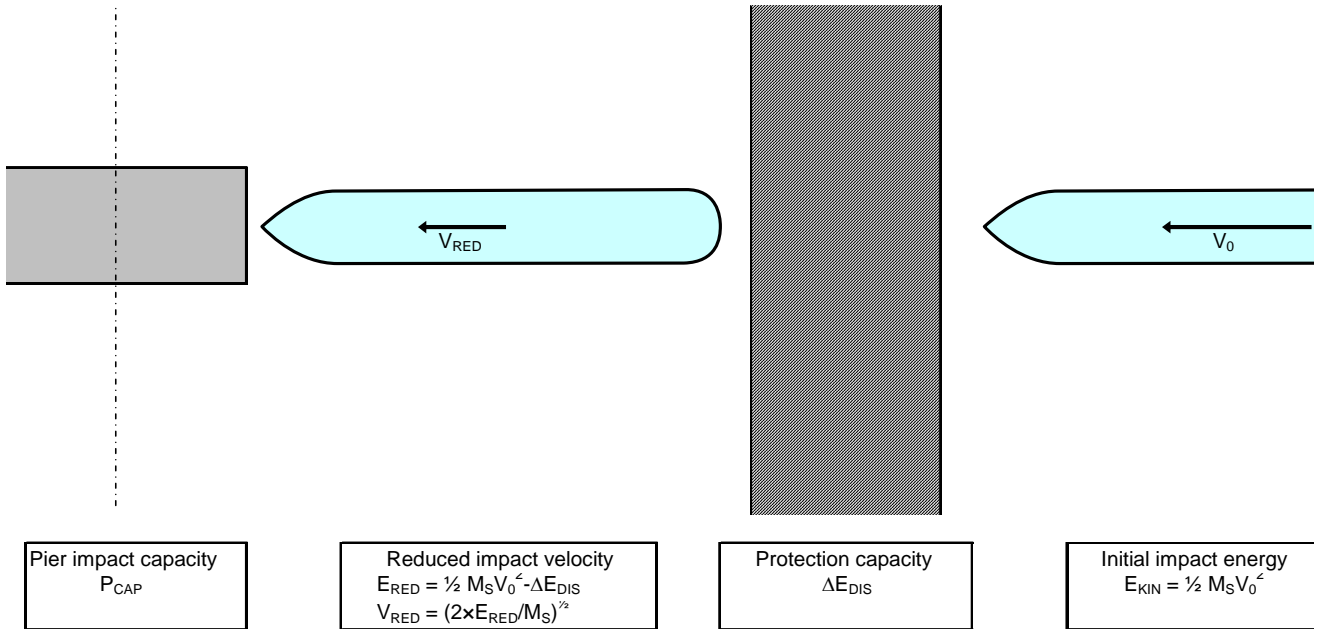


Figure 5: Reduction in impact energy due to the external protection.

If the initial impact energy E_{KIN} of the considered ship is larger than the dissipation capability of the external protection E_{DIS} , the ship is considered able to pass through the protection and reach the pier with a reduced impact speed V_{RED} . This approach makes it possible to control the risk of collapse of the pier both through adjustment of the external protection capacity and through adjustment of the pier impact capacity.

CALCULATION EXAMPLES

Illustration of the use of the AASHTO risk calculation method to determine impact requirements are given in the following. The example calculations are referred to as Strategy 1, 2 and 3.

The ship traffic corresponding to the future scenario (Year 2040) is used in the examples and a two-way traffic arrangement¹ is assumed.

Strategy 1: AASHTO Method with PPR 100,000 DWT requirements

A risk calculation based on the AASHTO method and using protection impact capacity for the 8 protected piers corresponding to the 100,000 DWT design vessel specified in the PPR will result in a calculated risk equal to zero - see Table 1.

¹ Two-way traffic arrangement implies that half of the width of the navigation channel is a dedicated lane for incoming ships and the other half is a dedicated lane for outgoing ships. This separated arrangement gives the largest exposure to impact in the risk analysis and the results are therefore conservative in comparison to other possible lane arrangements.

Pier ID	Pier Impact Capacity MN	Protection, Collision Energy MNm	Girder Impact Capacity MN	Pier Collapse Frequency [$\times 10^{-4}$ /year]	Girder Collapse Frequency [$\times 10^{-4}$ /year]	Total Collapse Frequency [$\times 10^{-4}$ /year]	
W10	2.0		2.0				1e-7 1e-6 1e-5 1e-4
W9	2.0		2.0	Not included in AASHTO			
W8	2.0		2.0	risk calculation			
W7	2.0		2.0				
W6	2.0		2.0				
W5	2.0		2.0	(distance to lane > $3 \times L_{OA}$)			
W4	12.0	90.0	2.0		0.00	0.00	
W3	25.0	210.0	2.0				
W2	25.0	380.0	2.0				
W1	70.0	1,250.0	2.0				
E1	70.0	1,250.0	2.0				
E2	25.0	380.0	2.0				
E3	25.0	210.0	2.0				
E4	12.0	90.0	2.0		0.00	0.00	
E5	2.0		2.0	Not included in AASHTO			
E6	2.0		2.0				
E7	2.0		2.0	risk calculation			
E8	2.0		2.0				
E9	2.0		2.0				
E10	2.0		2.0	(distance to lane > $3 \times L_{OA}$)			
Main bridge		E1-E4, W1-W4			0.00	0.00	1e-7 1e-6 1e-5 1e-4
Approach bridge		E5-E10, W5-W10					
Total					0.00	0.00	

Piers Girders

Table 1: Design requirements and associated risk estimates for Strategy 1.

The zero risk result suggests the selection of a 100,000 DWT ship as design vessel for all protection to be conservative. However, this choice may likely be guided by the expectation that the risk associated with the approach spans and/or the risk associated with oblique impacts, both of which are not included in the AASHTO calculation, in combination will contribute such that the acceptable risk becomes 10^{-4} per year.

Strategy 2: AASHTO Method

To illustrate the intended application of the AASHTO risk based design method a design optimization of the design of the protection for the 8 protected piers has been performed. The impact capacities of the protected piers themselves are assumed as shown in Table 2.

The risk optimized requirements to the impact protection and the associated collapse risk is summarized in Table 3.

Pier	Impact capacity
E1, W1	70 MN
E2, W2	25 MN
E3, W3	25 MN
E4, W4	12 MN

Table 2: Impact capacities of protected piers.

Pier ID	Pier Impact Capacity MN	Protection, Collision Energy MNm	Girder Impact Capacity MN	Pier Collapse Frequency [$\times 10^{-4}$ /year]	Girder Collapse Frequency [$\times 10^{-4}$ /year]	Total Collapse Frequency [$\times 10^{-4}$ /year]	
W10	2.0		2.0				1e-7 1e-6 1e-5 1e-4
W9	2.0		2.0	Not included in AASHTO			
W8	2.0		2.0				
W7	2.0		2.0	risk calculation			
W6	2.0		2.0				
W5	2.0		2.0	(distance to lane $> 3 \times L_{OA}$)			
W4	12.0	35.0	2.0	0.11	0.00	0.11	
W3	25.0	95.0	2.0	0.13		0.13	
W2	25.0	265.0	2.0	0.14		0.14	
W1	70.0	940.0	2.0	0.12		0.12	
E1	70.0	940.0	2.0	0.12		0.12	
E2	25.0	265.0	2.0	0.14		0.14	
E3	25.0	95.0	2.0	0.13		0.13	
E4	12.0	35.0	2.0	0.11	0.00	0.11	
E5	2.0		2.0				
E6	2.0		2.0	Not included in AASHTO			
E7	2.0		2.0				
E8	2.0		2.0	risk calculation			
E9	2.0		2.0				
E10	2.0		2.0	(distance to lane $> 3 \times L_{OA}$)			
Main bridge		E1-E4, W1-W4		1.00	0.00	1.00	1e-7 1e-6 1e-5 1e-4
Approach bridge		E5-E10, W5-W10					
Total				1.00	0.00	1.00	

Piers Girders

Table 3: Design requirements and associated risk estimates for Strategy 2.

For comparison with the PPR design ship of 100,000 DWT the optimized protection energy dissipations given in Table 3 are converted to equivalent design ships as shown in Table 4:

Pier	Protection Energy Dissipation	Corresponding Design Ship Size
E1, W1	940 MJ	75,000 DWT
E2, W2	265 MJ	70,000 DWT
E3, W3	95 MJ	45,000 DWT
E4, W4	35 MJ	39,000 DWT

Table 4: Equivalent Design Ship Sizes

It is seen that the straight forward application of the AASHTO risk analysis methodology to define the protection requirements leads to smaller design ships than the 100,000 DWT specified in the PPR.

Strategy 3: AASHTO Method including Approach Spans

Recognizing that it may have been the intention of the PPR (though not explicitly stated) that the AASTHO method is extended to also cover the approach span piers, a risk calculation has been performed that includes the calculated risk for the approach piers. For this calculation the actual impact capacity of the approach piers is assumed to be 12MN and not subject to risk optimization. The risk contributions from the unprotected approach piers will reduce the residual risk left for distribution between the protected piers, and the design requirements for these will then have to be increased compared to Strategy 2 in order to meet the requirement of a collapse risk less than 10^{-4} per year.

The resulting protection requirements and associated risk distribution is presented in Table 5.

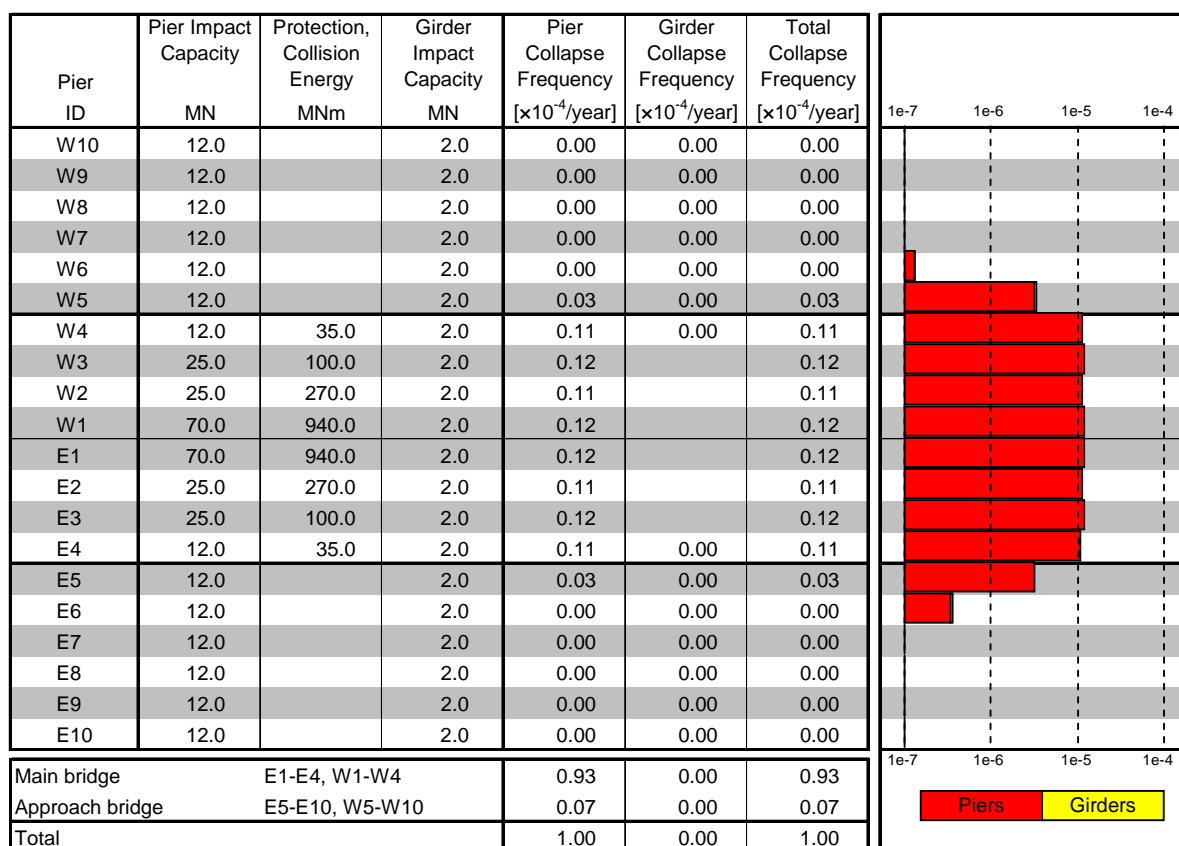


Table 5: Design requirements and associated risk estimates for Strategy 3.

The design ship sizes corresponding to the optimized impact dissipation energies given in Table 5 are shown in Table 6.

Pier	Protection Energy Dissipation	Corresponding Design Ship Size
E1, W1	940 MJ	75,000 DWT
E2, W2	270 MJ	71,000 DWT
E3, W3	100 MJ	48,000 DWT
E4, W4	35 MJ	39,000 DWT

Table 6: Optimized impact dissipation energies.

Inclusion of the risk contribution from the approach piers thus only cause a slight increase the design requirements to the protection.

CONCLUSIONS - CALCULATION STRATEGY

On the basis of the method discussion and calculated examples above the PPR are interpreted as follows for use in the detailed design of the ship impact protection:

- the 100,000DWT design ship combined with the AASHTO formula for the impact velocity that is given in the PPR for design of the impact protection is used for design of the protection in the primary impact direction parallel with the navigational channel
- the risk contributions from the unprotected approach piers calculated in Strategy 3 (0.07×10^{-4} per year) shall be taken into account.
- The approach span piers (E10-E5 and W5-W10) are located outside of the distance 3.0times LOA on each side of the inbound and outbound vessel transit centre line paths. Therefore, according to the AASHTO LRFD 3.14.5, the annual frequency of collapse for the approach span piers will not be considered.
- the remaining acceptable risk (1.0×10^{-4} per year) is used in a risk based optimization of the protection arrangement that considers impact scenarios that are not parallel with the navigational channel

DETAILED RISK ANALYSIS

The AASHTO method can be used to determine the necessary energy dissipation capacity of the impact protection. But evaluation and optimization of the layout or arrangement of the external protection cannot be made solely on the basis of the AASHTO method. This task requires that the probabilistic representation of ship impact scenarios is extended to consider different (all) impact directions - their relative probability and associated impact velocity.

IMPACT SCENARIO MODEL

The AASHTO risk method provides a probabilistic model using one or more normal distribution functions that a ship is heading towards a point on the bridge alignment. Supplementing this model with a probabilistic description of the impact angle will provide the detailed model needed for evaluation of the protection layout. The two models for impact position and direction are for simplicity assumed to be independent.

The continuous spectrum of possible impact angles is represented in terms of 19 explicit directions from 0° to 180° and each impact angle is associated with a relative probability and a relative impact velocity, see Figure 6.

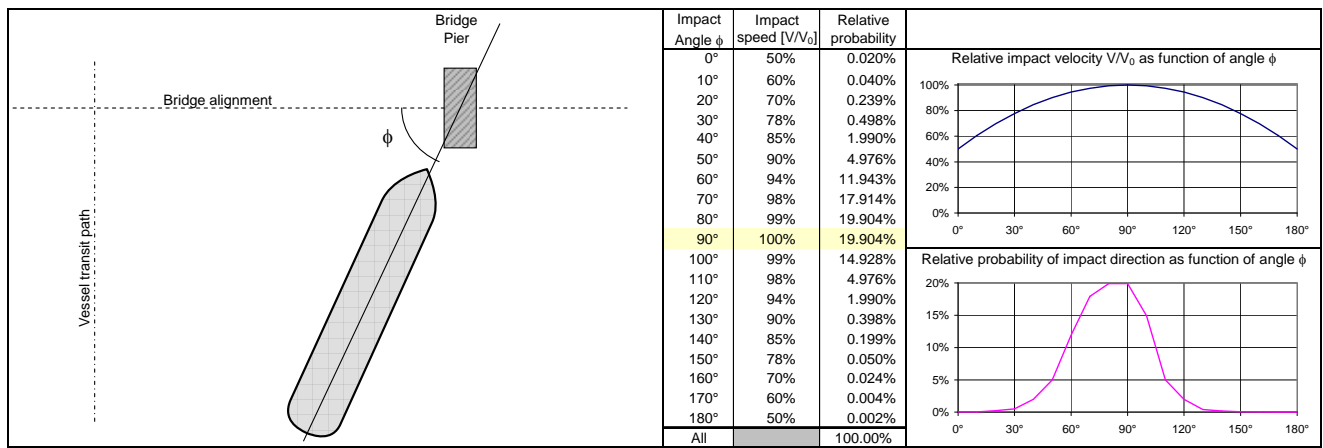


Figure 6: Probabilistic modelling of pier impact scenarios.

The impact velocity is expressed relative to the design impact velocity specified in the AASHTO method and thus adopts the associated moderation of the design impact velocity away from the navigation channel. The variation of the relative impact velocity with the impact angle is based on a quadratic form being 1.0 V_0 for 90° angle (parallel with the transit path) and 0.5 V_0 for impacts parallel with the bridge alignment (0° or 180°). These two fix-points for the design impact velocity in accordance with from the AASHTO method.

The variation of the relative probability of different impact angles has been judged on basis of the following logical inferences:

- impact parallel with the transit vessel path will be most likely
- impact perpendicular to and away from the vessel transit path is infrequent compared to parallel impacts and the relative impact probability is set 3 decades below the parallel impact
- impact perpendicular to and towards the vessel transit path is considered even more infrequent than the above perpendicular impact and is set 4 decades below the probability of parallel impact

The smooth variation between these fix-points has been applied as shown in Figure 6.

IMPACT SCENARIO EVALUATION

An important element in the evaluation of detailed impact scenarios is the ability to simulate and evaluate the interaction between the impacting ship and protection arrangement. A computation calculation model has been developed in Excel that simulates the motion of a stiff body ship as it impacts protection cells in a cell arrangement. The model includes the following elements:

- Mass and geometry of each ship in the population
(117 different ships are used to represent the actual ship traffic)
- Non-linear force-indentation of the ship bow

- For each individual cell:
 - Location
 - Geometry
 - Effective mass
 - Non-linear force-indentation
 - Non-linear force-translation
 - Ultimate deformation
- Geometric outline of the protected piers

A snapshot of a 45° impact is shown in Figure 7.

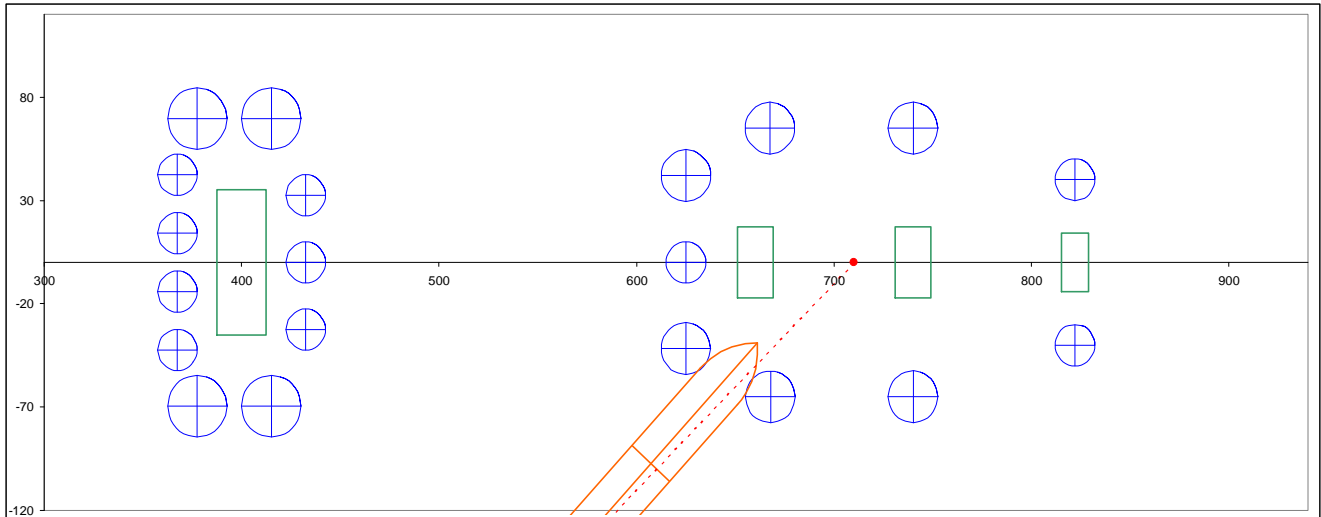


Figure 7: Snapshot of simulation of a 45° impact.

The simulation is using explicit integration of the dynamic equations of motion of ship and cells and is continued until:

- contact with a pier is detected
- the remaining kinetic energy of the ship is negligible
- or the ship no longer is on a course towards the protected piers
- When a simulation results in contact with a pier the frequency of that particular scenario is calculated and the impact force is determined for comparison with the impact capacity of the pier.
- The simulations are performed systematically for:
 - each ship in the population
 - appropriately spaced impact points along the bridge alignment
 - representative impact directions

For each impact scenario leading to contact the associated probability of pier collapse is calculated and these contributions are added to form the total probability of bridge collapse.

PROTECTION DESIGN OPTIMIZATION

The optimization of the protection arrangement has to be based on trial and evaluation: a protection arrangement is suggested (size and location of cells), the performance is evaluated from the impact simulations and the combined collapse frequency of the protected piers is calculated. If the combined collapse frequency is below the target the protection arrangement can be reduced and if above the target the arrangement has to be strengthened. Adjustments of the protection arrangement can be guided by inspection of the scenarios contributing to the collapse risk.

CONCLUSION - RISK BASED DESIGN

Pertinent detailed aspects of the detailed design of the ship impact protection of the Incheon Bridge have been discussed with reference to the Project Performance Requirements (PPR) and the AASHTO methodology for risk based design of bridges against ship impact. Evaluations are made concerning minor uncertainties regarding

the appropriate methodology to use in the detailed design and relevant supplementing basis and necessary tools are described.

In conclusion the detailed design of the ship impact protection of the 8 piers (E1-E4 and W1-W4) will be carried out as follows:

- for impacts parallel to the navigation channel the capacity is determined using the PPR design vessel of 100,000 DWT
- for impacts in other directions a detailed risk analysis is made based on numerical simulation of the ship-protection interaction and using a target risk of collapse of the protected piers of 1.0×10^{-4} per year

STRUCTURAL MODELLING

The verification of the stopping capacities involves significant deformation and non-linearity and therefore the compressive 3-dimensional FE-program ABAQUS was used to document dissipation capacity of the dolphin structure. The dynamic module ABAQUS/Explicit can model the dynamic behaviour of the ship impact. The modelling was used to access only the extreme load case of a ship impact to:

- Determine the stopping capacity of the dolphins for specific impact load cases
- Determine the response characteristics of the dolphin for use in the 2D ship impact simulation
- To provide local stresses and strain data for verification of the integrity of the dolphin during the ship impact

The 3D FEM model includes the soil outside and below the dolphin, sheet pile with concrete cap and the crushed rock fill inside the dolphin. Figure 8 below shows the FEM model. The blue box in the figure represents the bulb of the ship. At the back of the box is a weight node with the dead weight of a 100,000 DWT ship. Only half of the 3D dolphin is modelled, using loading symmetry. At the back of the box is a mass node with half of the dead weight of a 100,000 DWT ship.

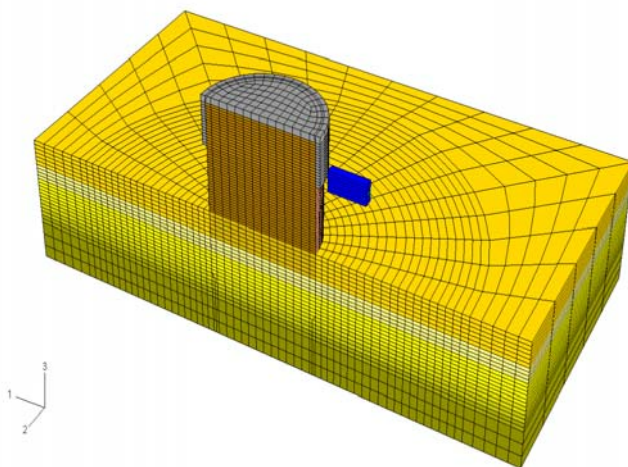


Figure 8: Structural FEM-Model (unloaded).

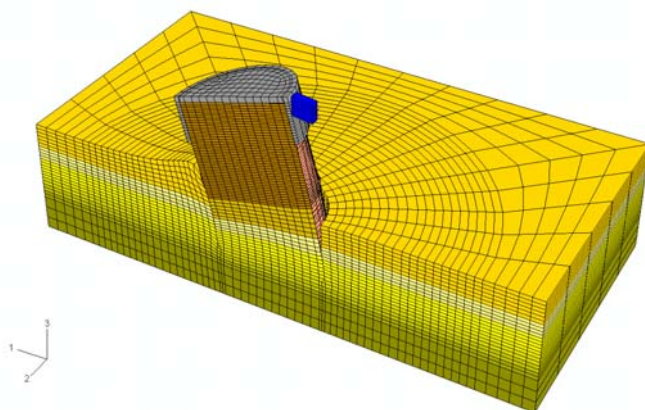


Figure 9: Structural FEM-Model (loaded).

The sheet pile and concrete cap are modelled as structural elements with continuum shell elements. The sheet pile has properties giving it membrane and bending behaviour as the actual used sheet pile. The concrete cap is modelled as concrete and reinforcement is included. All soil layers are modelled with solid elements. The extended Drucker Prager was used as material model for soils.

Adaptive remeshing, where the mesh remeshes elements in areas with very large deformation, was used to improve mesh quality throughout the entire impact analyses.

Using the general contact method in ABAQUS/Explicit the interaction soil-sheet pile, soil-concrete cap, sheet pile-concrete cap and dolphin ship bulb are modelled with hard contact and coulomb friction. No negative pressure was allowed in the interface, allowing separation.

After an initial step to get an initial stress state in the soil and structure, the mass node is accelerated to the design velocity of the ship. The node is then given free, with the required velocity and mass. Figure 9 depicts the deformation after the ship has impacted the protection dolphin.

Automatic model generation was established to make the models with different dolphin diameters and different soil profiles easy. Also automatic output generation was established, so that output from the modelling easily could be

used in the 2D impact simulations.

The main output of the 3D FEM modelling was energy output. The modelling showed that the kinetic energy of the impacting ship will be absorbed by dissipation from crushing of ship bow, dissipation of local crushing of dolphin side and dissipation due to global overturning of the dolphin. Figure 10 is an illustration of the energy dissipation from an Ø25m dolphin during the impacting of a ship with velocity 5.14m/s.

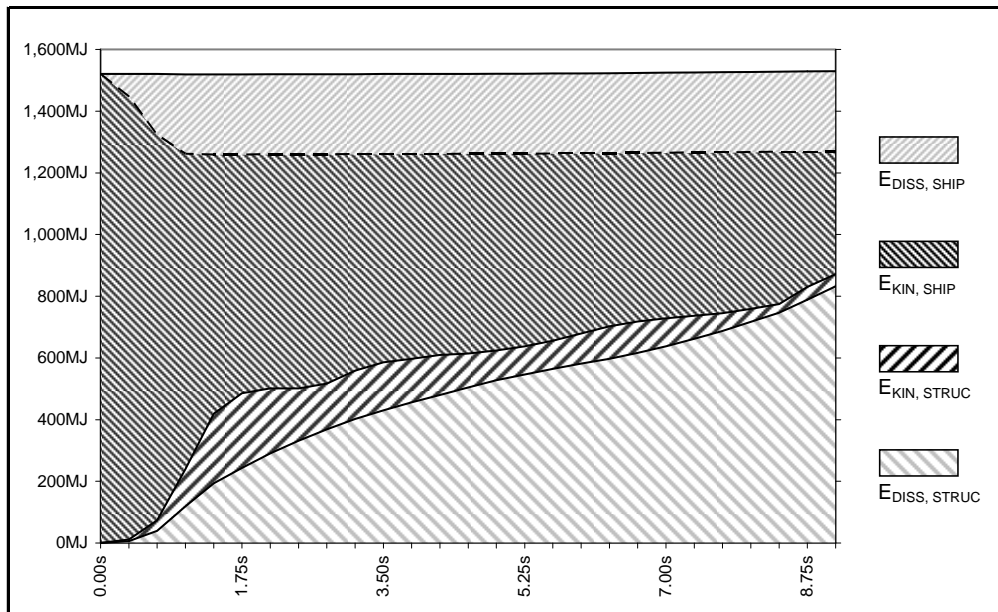


Figure 10: Energy dissipation plotted against time.

The response characteristics of all the FEM models were approximated for use in the risked simulation analysis of the protection layout. The response characteristics determined for each model includes:

- force-indentation characteristics related to the local deformation at the impact area
- force-overturning characteristics related to the global overturning of the dolphin
- the equivalent mass associated with the global overturning

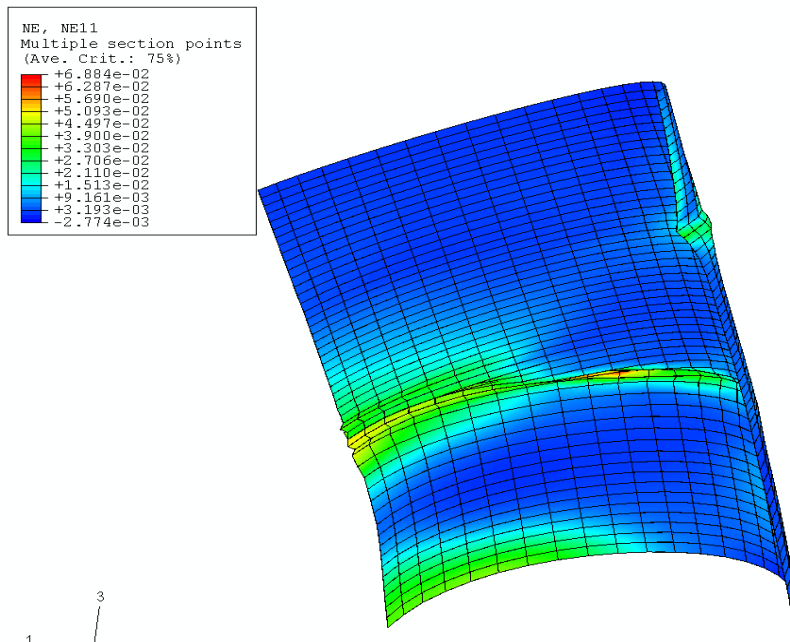


Figure 11: Sheet Pile wall of the Protection Structure.

This provides the necessary input for the 2D impact simulations. Because the sheet pile is modelled as a structural element, strain, stresses and bending moments can be shown. Figure 11 shows the hoop strain in the sheet pile of a Ø25m protection dolphin. The hoop strain is important information for choosing a right sheet pile type with locks that can take the shown strain.

For strength limit state and the extreme seismic load case was evaluated with the 2D geotechnical finite element programme PLAXIS. Plane strain is assumed and the width of the structural model is approximated with the side length of a square with the area as the sectional area of the circular dolphins. The material model used was the

simple Mohr-Coulomb material model. The model includes the sheet pile with plates and with concrete cap as stiff elastic soil elements. Figure 12 shows the 2D PLAXIS model applied.

Using the ϕ -c reduction method in PLAXIS, where strength properties, friction and cohesion, are reduced until failure.

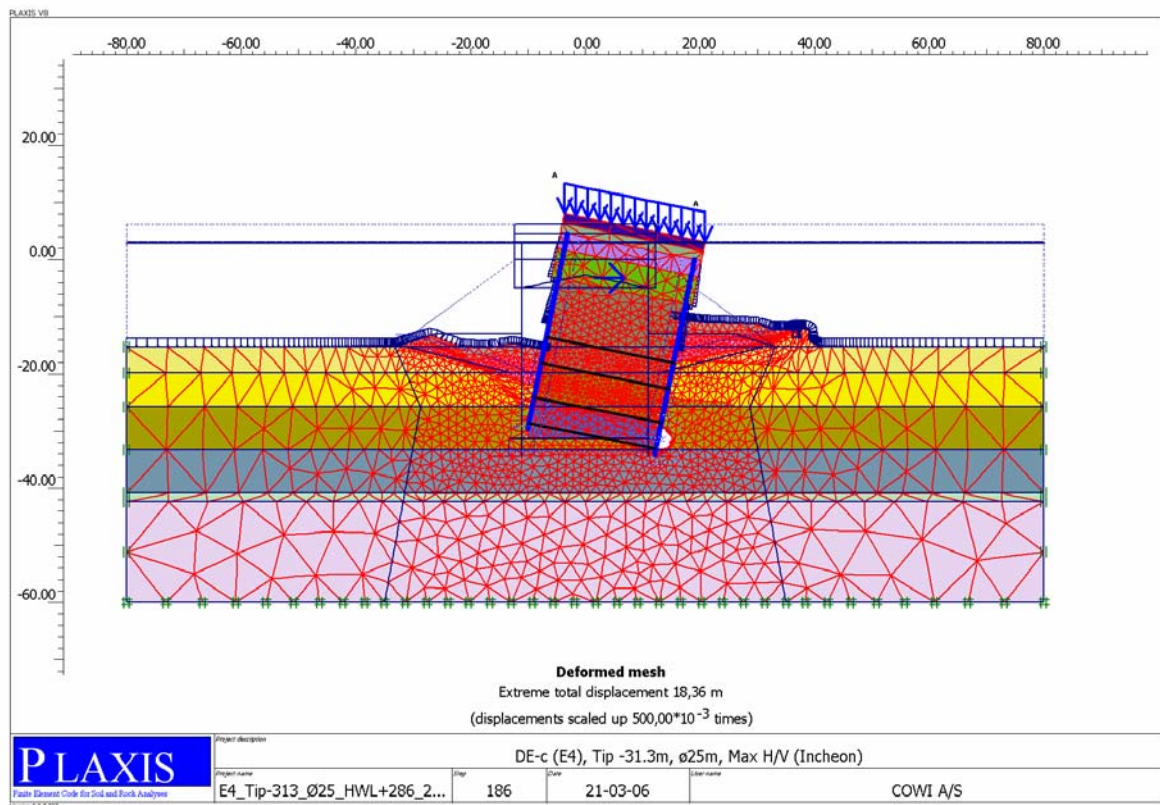


Figure 12: 2D Finite Element Model - PLAXIS.

The ϕ -c reduction method produces Figure 13 where M_{sf} is shown against deformation of the centre of the top of the dolphin. M_{sf} indicates the partial factor the soils strength properties. The required partial factor is 1.0 but due to the fact that FEM is upper limit method, result depending on mesh density, the required M_{sf} shall be added 5% of the required partial factor. Therefore the $M_{sf}=1.05$ shall be obtained in the PLAXIS modelling.

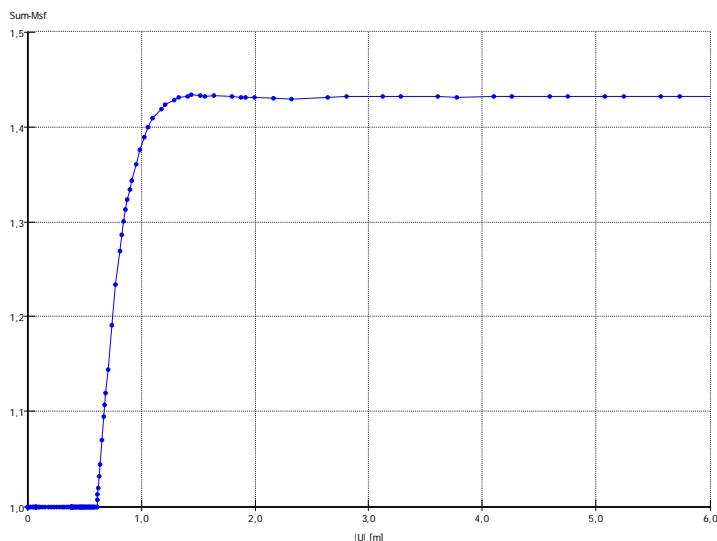


Figure 13: M_{sf} plotted against deformation.

CONCLUSION - STRUCTURAL MODELLING

The output of the advanced dynamic 3D ABAQUS FE-analysis was used as input to the 2D impact simulations and provided detailed structural strains and stresses during the ship impact scenario. The 2D PLAXIS FE-analysis showed sufficient safety in service limit state and the extreme event limit state - earthquake.

GEOTECHNICAL CONDITIONS

GENERAL GEOTECHNICAL CONDITIONS

A number of geotechnical boreholes have been carried out along the bridge alignment in order to provide sufficient data for the detailed design of the foundation of the individual bridge structures. In order of decreasing importance the evaluation of the stratigraphy and soil conditions for the 44 Ship Impact Protection Structures (protecting of pier E1 to E4 and W1 to W4) are based on: specific SIP boreholes, boreholes for the Cable Stayed Bridge, boreholes for the Approach Bridges and other boreholes. The layout of the Ship Protection Structures is shown in Figure 14.

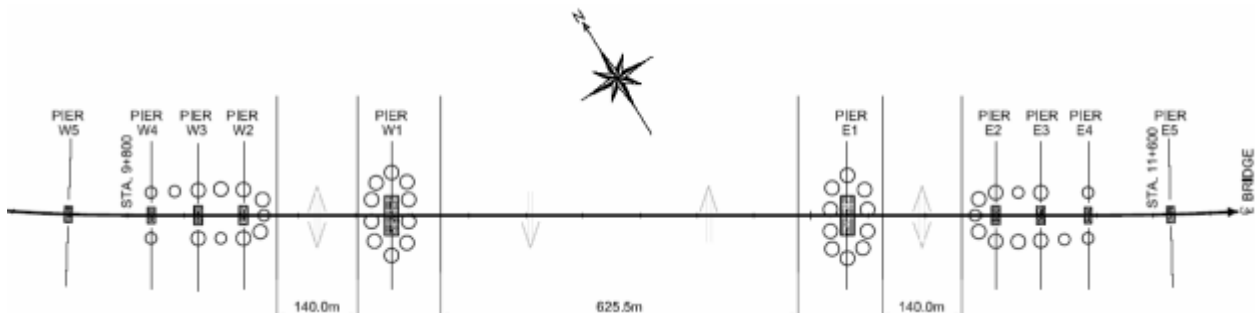


Figure 14: Layout of piers and Ship Impact Protection Structures.

Based on the boreholes in combination with relevant laboratory testing the overall stratigraphy may be summarised as (values in brackets are total thickness):

- Marine deposits (Clay, Silt and Sand) [15.5 to 25.5 m]
- Residual soils [0 to 16.5 m]
- Weathered rock [2.5 to minimum 11.5 m]
- Soft rock

The marine deposits consist of layers of clay, silt and sand (roughly in the ratio 0.27/0.13/0.50) in irregular sequence and considerable variation in thickness and properties in-between the location of the Ship Protection Structures.

The clay layers are generally clays of low plasticity ranging in stiffness from very soft to hard (SPT- N_{60} values from 0 to 68). The silt layers range from pure silt to sandy and clayey silt, non-plastic to low plasticity and very soft to hard (SPT- N_{60} values from 0 to 44). The sand deposits range from silty sand to medium grained sands, often poorly graded, and is found in loose to very dense packing (SPT- N_{60} values from 0 to 100).

The residual soils consist of silty sand with a small amount of rock fragments and show a gradual transition to the weathered rock below. The deposit ranges from medium dense to dense (SPT- N_{60} values from 21 to 125).

The thickness of the weathered rock has in general not been found as only a few of the boreholes have penetrated this stratum. The Biotite Granite is highly (D4) to completely (D5) weathered rock but shows high strength (SPT- N_{60} values from 107 to 750).

All SPT-N values exceeding 50 are extrapolated values. For ease of interpretation the distinction between residual soil and weathered rock was taken as SPT-N = 100.

The soft rock underlying the whole area ranges from strong to weak rock with fractured/weak zones and cross joints. The rock is Biotite Granite moderately (D3) to completely weathered (D5) and the RQD values range from 0 to 88% with a total core recovery of TCR = 60 to 100%. This deposit plays no role for the resistance of the Ship Impact Structures.

TYPICAL SOIL PROFILES

Due to the highly variable soil conditions (variation from borehole to borehole) all boreholes in the vicinity of the Ship Protection Structures were considered in order to produce representative design profiles for the individual Ship Protection Structures. A total of nine different design profiles were developed for the design of the 44 Ship Impact structures for both drained and undrained conditions.

The design profiles were developed based on the available field information of SPT testing in soils and pressure meter testing in the soft rock. The field tests were supplemented by laboratory classification tests, 10 triaxial UU

test and three triaxial CU tests. Deformation parameters were inferred from well winnowed international empirical correlation rules based on the classification tests.

One of the crucial assumptions made were that the soil types could be distinguished based on the plasticity index. This has bearing on the inherent soil behaviour (fine grained or coarse grained) and on the evaluation of undrained shear strength based on factoring of the SPT-N values and hence a cautious approach was adopted in order to provide a robust design.

Subsequently the design profiles (stratigraphy and strength/deformation parameters) were checked versus 31 additional, dedicated Ship Impact Structure boreholes carried out after the completion of the design in order to enhance the estimation of necessary excavation levels, stratigraphy, strength and particular features relevant for design and the necessary tip level for the sheet piles in the protection structures. In connection with the classification testing associated with the additional boreholes 10 series of four direct simple shear tests were carried out on non-plastic silt or silty sand.

By and large the adopted design profiles were shown to be adequate with the exception of two structures where modifications had to be made at a late stage. The cautious approach adopted in the original evaluation of the design parameters were borne out by the new investigation results where the soil description terms were evaluated to differ slightly from the original descriptions (clayey silt corresponded to silty/sandy clay in the first round of investigations).

IMPORTANT GEOTECHNICAL CONSIDERATIONS

Due to the inherent nature of the ship impact (which takes place very rapidly) the undrained shear strength of the sand and sandy silt deposits is very relevant. However, very few laboratory results were available to evaluate this parameter. Based on two consolidated undrained triaxial tests on sandy silt it was concluded that the soil would exhibit negative pore pressure development (equivalent to dilation in drained conditions). Hence, these deposits were conservatively assumed to be drained in the design.

In general the choice of soil profiles entailed a balanced evaluation of the most onerous combinations in terms of bearing capacity, side resistance and settlements. Thus the characteristic soil profiles are associated with choices of maximum possible amount of clay in particular with depth.

In order to function according to the design much attention was given to the fill inside the Dolphins and to the associated water level fluctuations in the fill material. Thus, the interaction between the vertical effective stress and the wall friction was studied in detail.

Both analytical and numerical models were used for the analysis of the different limit states, strength limit state, service limit state and extreme limit states (earthquake and ship impact).

In order to be able to treat the large number of design situations and soil profiles the bearing capacity calculations were by and large checked using the 2D PLAXIS finite element programme. It was evaluated that this would provide a conservative estimate compared with the much more cumbersome and time consuming 3D modelling.

However, for the ship impact it was considered imperative to be able to quantify and qualify the behaviour by 3D dynamic analyses using ABAQUS 3D fine element software. In order to gain confidence in the modelling of this crucial part of the Ship Impact Structures it was decided to carry out physical modelling at reduced scale in a centrifuge. The rationale was that modelling of models in the centrifuge (using simplified geometry and soil stratigraphy) would provide:

- insight in the relevant failure modes
- the soil-structure-water interaction in the event of a real time modelled impact and
- provide the necessary backbone data for verification of the numerical 3D modelling with "real" soil stratification and geometry.

The length of the flat sheet pile driven for the Dolphin Ship Protection Structures were by and large determined by the occurrence of soil strata of sand/residual soil/weathered rock with SPT-N values exceeding 40.

PHYSICAL MODEL TESTING

MODELLING CONSIDERATIONS

The Dolphins are sacrificial structures that will be partly or fully destroyed in the event of a severe ship impact. Thus, the stopping capability of the Dolphins involves very large deformations, non-linear soil behaviour and dynamic soil-structure interaction. Finite element modelling using ABAQUS software has generally been used to document dissipation capability of the structures. Due to unusual inclusion of structural resistance at such large deformations the PPR requirements necessitated experimental verification.

To realistically model the 20 to 30 m diameter structures it was realised that reduced scale had to be used. The presence of large quantities of sand or other coarse grained material necessitated testing in a geotechnical centrifuge to allow correct representation of the soil stresses at reduced model scale.

The main objectives of the centrifuge testing programme were to provide experimental results for:

- the global quasi-static force response of the dolphins for direct comparison with the response predicted by 3D FEM analysis
- the global response in dynamic impact scenarios for comparison with the quasi-static experimental results
- the local force-indentation relationship for deep impact causing local indentation and even damage of the sheet piles

The laboratory selected to carry out the physical model testing was GeoDelft (now Deltares) with access to one of the largest commercially available centrifuges world wide. The limitations on geometric extent and force capability of the centrifuge resulted in a scaling of 1:200 of the 20 m and 30 m diameter anticipated dolphin structures subjected to an acceleration of 200 g.

The physical model tests considered the behaviour of a single circular dolphin with very simplified albeit realistic soil stratification using homogeneous layers. Thus, emphasis was placed on achieving reproducible testing conditions allowing high credibility of output and well defined conditions for comparison with the numerical 3D calculations.

As the "real" Dolphin is reduced by a factor $n = 200$ the soil grains should in principle also be reduced at the same scale to obtain complete similarity for flow etc. However, this was not considered practical and hence a fine grained model sand, Baskarp sand, with a mean grain size of 0.15 mm was used in the centrifuge testing. A high viscosity fluid, rather than water, would reduce the flow velocity and counteract the missing scaling of the soil grains but this was on balance not considered necessary nor feasible.

CENTRIFUGE MODEL

An artist's impression of the centrifuge model of the dolphin is shown in Figure 15 and the idealized prototype of the 30 m diameter dolphin is shown in Figure 16.

The following approximations were made for the idealized dolphin:

- the top soil down to the sheet pile tip is medium dense Baskarp sand
- the bottom soil below the sheet pile tip is a very dense layer of Baskarp sand
- the material inside the dolphin is medium dense or very dense Baskarp sand
- the sheet pile wall consist of a cylindrical steel shell with the same (in model scale) bending and axial stiffness as the sheet piles in the prototype (The sheet pile structure is modelled using an etching technique to produce a variable thickness representing the additional cross sectional area and vertical bending stiffness of the locks). In the initial test an isotropic steel shell was used.
- the locks are "fixed"
- the top cap has no slope but the aluminium cap has the correct weight

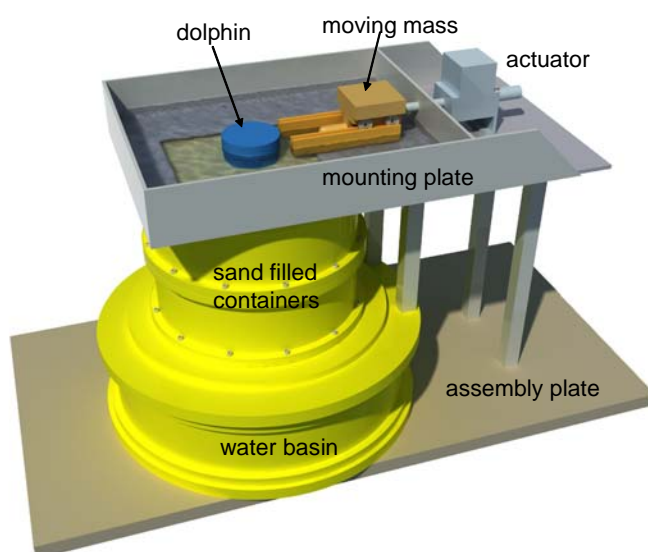


Figure 15: Artist's impression of the centrifuge model set-up (plunger from actuator either fixed in displacement control or pushing the moving mass to achieve impact control).

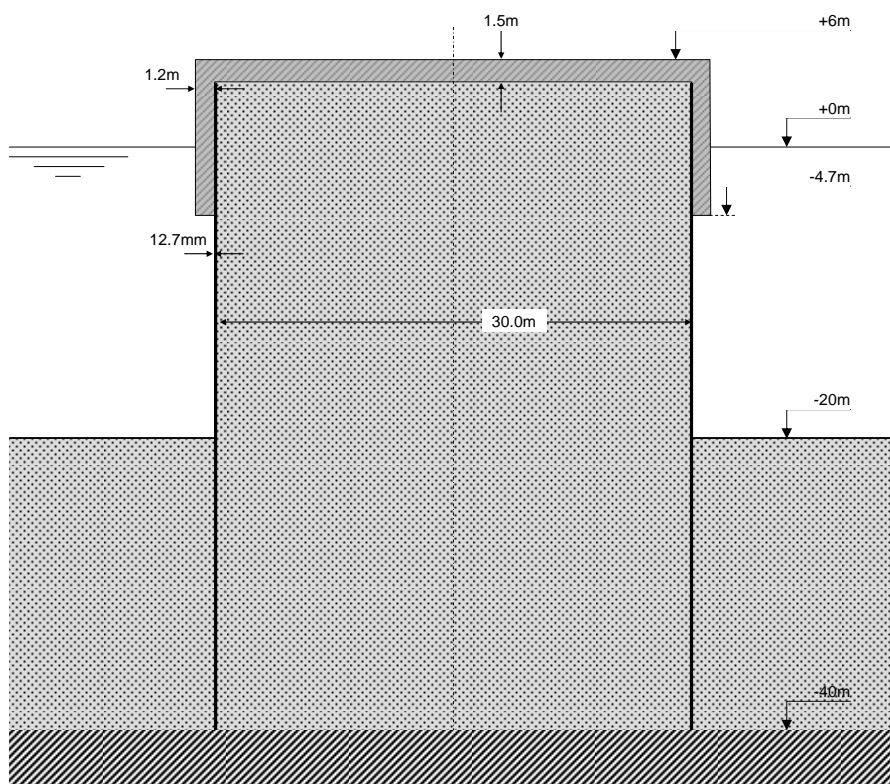


Figure 16: Idealized prototype with 30 m diameter dolphin.

The ship is modelled by a solid mass on wheels impacting the dolphin with a protruding cylinder with a spherical end representing a torpedo shaped bulb of a larger ship. The mass of the impacting object is 18.1 kg corresponding to a prototype mass of 145,000 tons.

The ship model is actuated by a hydraulic system that can

- move the ship forward at a constant slow speed (used for quasi-static tests)
- momentarily accelerate the ship to a speed up to 3.7 m/s allowing the ship to continue freely without contact with the plunger (used for dynamic impact tests)

The instrumentation included the force from the hydraulic system, the horizontal displacement of the mass, the x, y, z displacements of the top of the dolphin and pore pressure measurements in selected parts of the sand volume in the centreline of the dolphin. All data were logged to provide continuous record of data points with time.

TYPICAL TEST RESULTS

A total of 29 tests were carried out including repeated tests (for verification or to make up for unsuccessful tests) to provide data for 18 different test situations

- tests 1-4: Ideal rigid body dolphin for verification of test set-up (300 mm/h, 3 m/s)
- tests 5-8: Quasi-static tests with force in waterline 300 mm/h)
main results for comparison with FEM
- tests 9-12: Dynamic impact tests with force in waterline (3 m/s)
for comparison with quasi-static tests to quantify effect of dynamic impact
- tests 13-14: Dynamic tests at lower impact speed (2m/s and 1m /s)
for comparison with quasi-static tests to quantify effect of dynamic impact speed
- tests 15-16 Quasi-static tests at low level impact (-9 m in prototype)
test for effect of local indentation of sheet piles
- tests 17-18: Dynamic tests at low impact (-9 m in prototype)
test for effect of local indentation of sheet piles

The test results for tests 5 and 7 compared with the corresponding dynamic tests 9 and 11 are shown in Figure 17 and Figure 18.

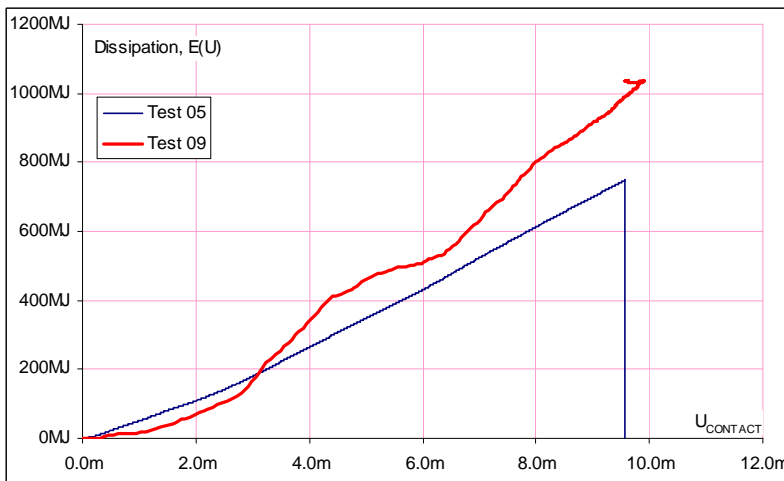


Figure 17: Energy transfer to dolphins as function of prototype displacement compared with quasi-static response.

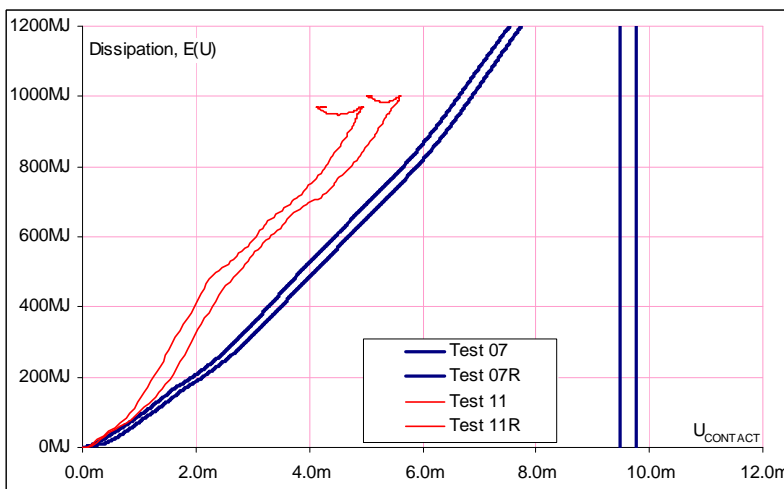


Figure 18: Energy transfer to dolphins as function of prototype displacement compared with quasi-static response.

CALIBRATION AGAINST 3D FEM RESULTS

The comparison between the centrifuge tests and the ABAQUS calculations for the quasi-static tests appear from Figure 19 for Test05 and Figure 20 for 07 and 07R.

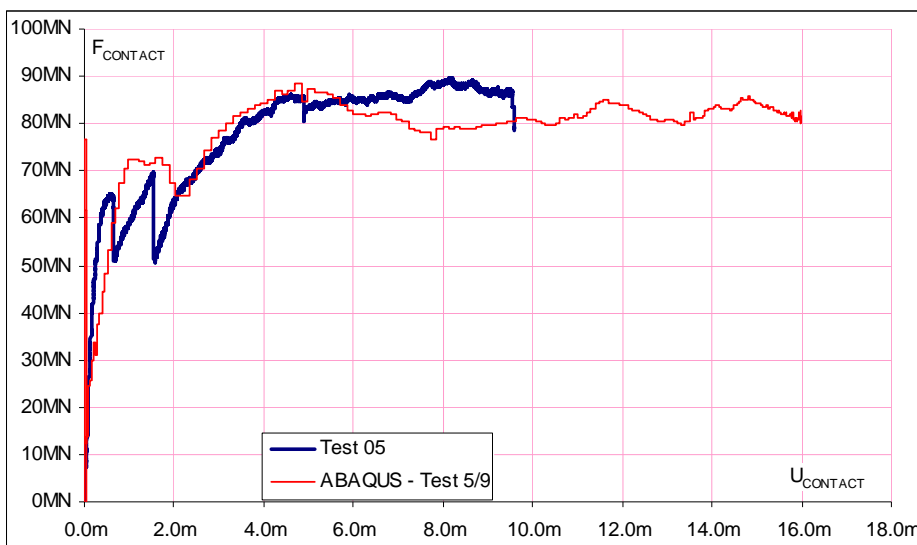


Figure 19: Force-displacement curves from centrifuge tests 05 compared with ABAQUS model.

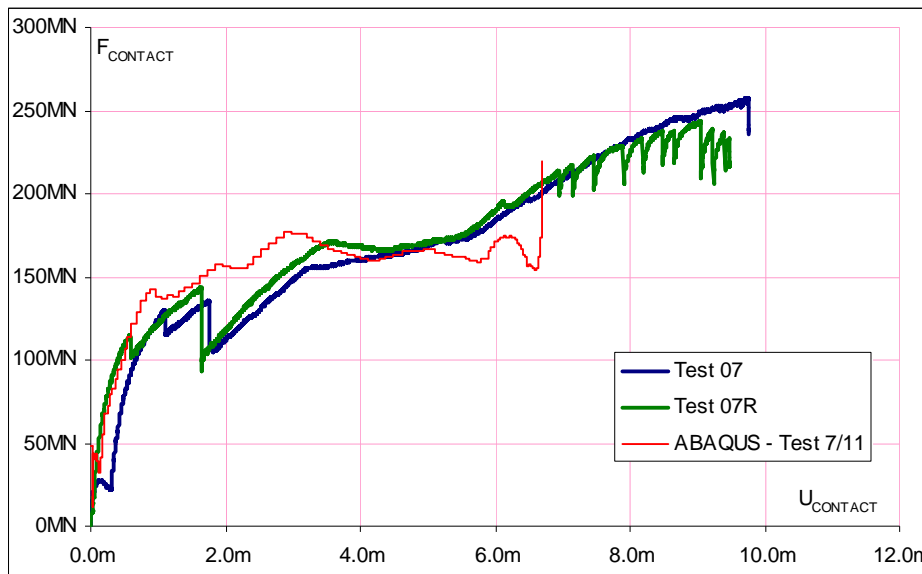


Figure 20: Force-displacement curves from centrifuge tests 07, 07R compared with ABAQUS model.

The result of the comparison between the physical model testing and the ABAQUS modelling is that the dissipation-displacement curves estimated with ABAQUS modelling is slightly above the experimental curves. However, the difference is of the same order of magnitude as the experimental accuracy and hence lends credibility to the possibility to model the quasi-static ship impact with confidence.

The dynamic and quasi-static experimental tests show a clear increase in dissipation in the dynamic test of the order of magnitude 38-50%. The tests show that increasing fill density leads to higher dissipation but no influence is observed from the dolphin diameter.

The overall conclusions of the centrifuge testing and the calibration of the 3D finite element modelling are that:

- ABAQUS 3D FEM has the ability to predict the force-displacement and dissipation response of the protection dolphins to an accuracy and detail required to lend credibility to modelling of more complex stratification for the actual dolphins
- the use of quasi-static response characteristics for design of the impact structures is conservative (30-50% higher dissipation capacity for dynamic impact)
- the strengthening of the dolphin in dynamic impact increases with increasing fill density
- low level impact with higher risk of steel membrane penetration is counteracted by the beneficial effect from reduced overturning moment (comparable or higher dissipation capacity than for higher level impact).

FINAL DESIGN OF SHIP PROTECTION STRUCTURES FOR THE INCHEON BRIDGE

Based upon the risk based design approach described in the paper it has been possible to utilize in a rational manner a:

- detailed modelling of realistic collision scenarios
- optimization of the layout of the ship protection structures
- advanced FE-modelling in modelling of structural capacity at large deformations
- advanced calibration of analytical/numerical results to results of physical models in a centrifuge
- detailed descriptions of soil strata at individual pier locations

to fully document a layout and design of the ship protection structures that are optimized to be cost-efficient yet achieving an acceptable risk level for the Incheon Fixed Link.